APPENDIX A
SHORING WALL DESIGN PARAMETERS

MEMORANDUM

DATE: February 12, 2002

TO: Drew Gangnes, SWMB

FROM: Michael Whelan, P.E., and John P. Laplante,
Hart Crowser, Inc.

RE: **Shoring Wall Design Parameters**
10 Broad Street
Seattle Washington
7018-02

This memorandum provides our recommended design parameters for the temporary shoring wall that will support excavation at the 10 Broad Street site, in Seattle, Washington.

At part of the redevelopment of the 10 Broad Street site, Hart Crowser is creating a plan for excavation of contaminated soils near Elliott Avenue. To facilitate this excavation, which will be on the order of about 18 feet deep, a temporary shoring wall will be required along Elliott Avenue. The ground surface elevation along Elliott Avenue is about 13 feet, City of Seattle datum, which will result in a planned bottom of excavation at elevation –5 feet.

Subsurface Conditions

Hart Crowser advanced one geotechnical boring at the location of the shoring wall. A copy of the boring log is attached for reference. The location of this boring is shown on Figure 1, Site and Exploration Plan. The following two major stratigraphic units were encountered:

Sandy, Gravelly CLAY FILL. This soil unit was generally medium stiff to stiff in consistency. The fill contains woody debris and other organics, and is present to a depth of about 15 feet.

Native SAND and GRAVEL. Beneath the sandy clay, slightly silty to very silty sand and gravel were encountered. This soil unit is very dense, with the exception of the upper 2 to 3 feet, which is medium dense. The sand and gravel extended to the bottom of our exploration, with increasing silt content at the bottom of the boring.

Groundwater. Groundwater was encountered at a depth of about 18 feet at the time of drilling, which corresponds to an elevation of about –5 feet, City of Seattle Datum. Periodic monitoring of groundwater levels in adjacent existing wells suggests that the static groundwater elevation in this area is at about elevation –3.5 feet, with little tidal influence. The shoring wall should be designed assuming that a groundwater elevation behind the wall of –2 feet, City of Seattle datum. This will result in an unbalanced hydrostatic head between the back of the wall and the base of the excavation, which should be accommodated by the shoring design. The need for dewatering to accomplish excavation and shoring construction must be evaluated, and it may be necessary to use well points or other means for temporary dewatering.

Soldier Pile Shoring Design Parameters

Temporary shoring along Elliott Avenue should be designed using an active (triangular) earth pressure distribution using equivalent fluid weights of 42 pounds per cubic foot (pcf) above the groundwater table, and 20 pcf below the groundwater table. Ultimate passive pressures should be calculated assuming a triangular passive pressure distribution with an equivalent passive fluid weight of 300 pcf. The upper 2 feet of soil at the base of the excavation should be ignored in the calculation of passive pressure, and a factor of safety of at least 1.5 should be applied to passive pressures. To account for the traffic surcharge along Elliott Avenue, 2 feet should be added to the height of retained soil (H). The active pressure should be extended to the base of the wall system (i.e., the bottom of the piles). Active earth pressures may be considered to act over one pile diameter below the base of the excavation. The recommended earth pressure distribution is shown graphically on Figure 2.

Wall deflections will need to be monitored during the time that the wall is in place. It may be possible to reduce the height of the shoring wall by using a cut slope at the top of the wall. This would have the potential advantage of lessening the required size of structural members in the wall and may enable the wall to be designed without tiebacks. The cut slope should be no steeper than 1H:1V. Account for such a slope in the shoring design by assuming that an effective additional height of soil, equal to one-half the slope height, is present behind the wall. See Figure 1. The maximum open cut height will be dictated by the available right-of-way that we can take. At most, this cut should be limited to 8 feet in height, however the available right-of-way may be much less.

If tiebacks are used, they should be tested during installation in accordance with the Tieback Testing Program presented in Attachment A. We have also included a Shoring Monitoring Program for the wall in Attachment B.

Soldier Pile Support Systems. Soldier piles must be designed to carry bending stresses. These stresses should be calculated from the active earth pressures. The soldier piles must be embedded deeply enough to provide kickout resistance for the portion of the wall below the excavation.

- Design soldier piles for bending using a uniform loading equivalent to 80 percent of the design earth pressure values. Analyze for shear using the total load.

- For design against kickout, compute the lateral resistance on the basis of passive pressure acting over twice the diameter of the soldier pile section or the pile spacing, whichever is less. This value should include a factor of safety of about 1.5.

- Embed soldier piles a minimum of 10 feet below the base of the excavation or to the required structural depth, whichever is greater.

Lagging. Loss of ground between the soldier piles is often prevented using timber lagging. The lagging is attached to the soldier pile and designed for some fraction of the applied pressure on the wall. The lagging will be designed using an applied lateral pressure of 80 percent of the design wall pressure.

Prompt and careful installation of lagging, particularly in areas of seepage and loose soils, is important to maintain the integrity of the excavation. Soil failure, sloughing, and loss of ground must be prevented. All voids behind the lagging should be backfilled thoroughly with a porous slurry or pea gravel. Lean concrete, foam, or other low permeability materials should not be allowed, because they could cause build-up of hydrostatic pressures behind the wall.

An extra lagging board or two should be installed above the shoring wall to provide a partial barrier for material that could fall into the excavation.

Attachments:
Figure 1 - Lateral Earth Pressures for Cantilever or Singly Braced Shoring
A - Tieback Testing Program
Figure A-1 - Key to Exploration Logs
Figure A-2 - Boring Log HC01-1
B - Shoring Monitoring Program

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Site and Exploration Plan

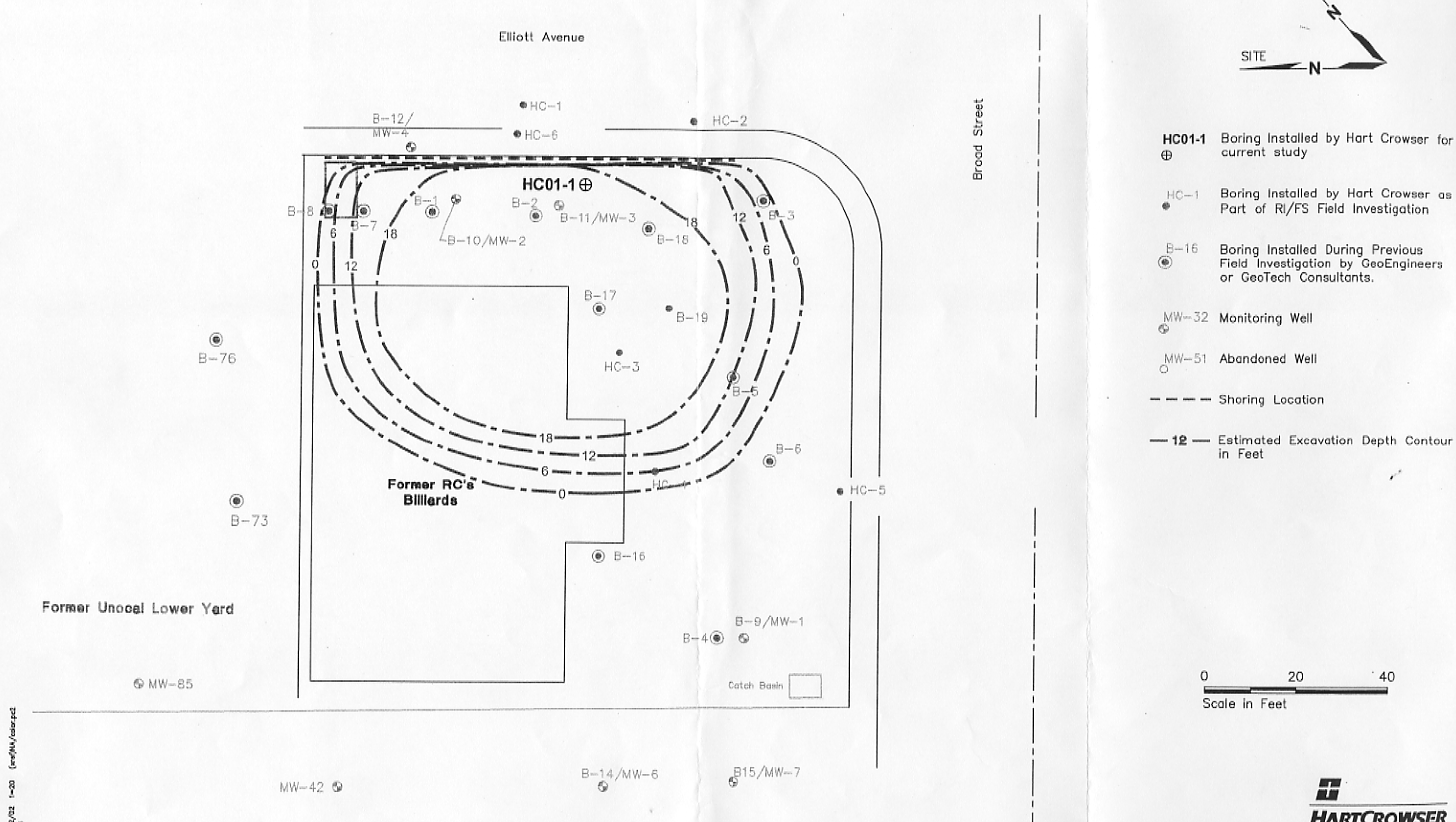

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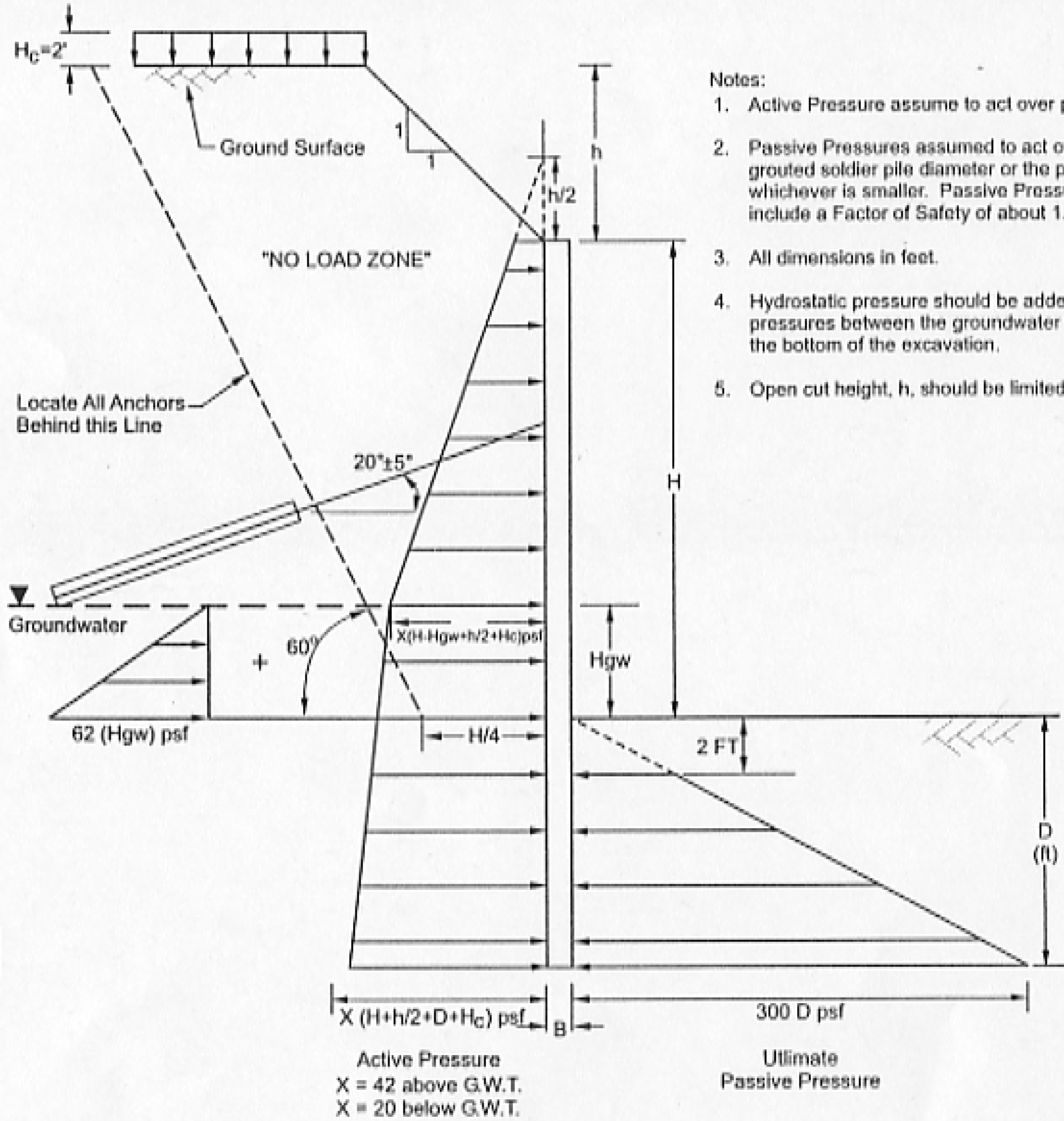
Figure 1

Lateral Earth Pressures for Cantilever or Singly Braced Shoring

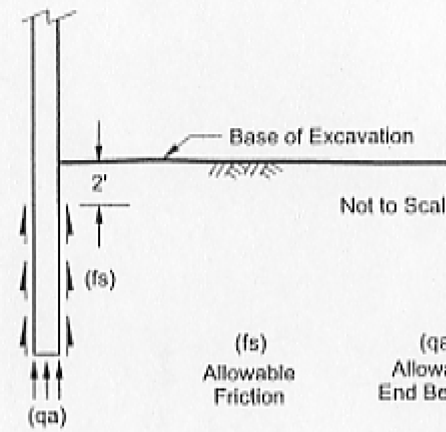
A. General Soil Pressures



Notes:

1. Active Pressure assume to act over pile spacing.
2. Passive Pressures assumed to act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller. Passive Pressures should include a Factor of Safety of about 1.5.
3. All dimensions in feet.
4. Hydrostatic pressure should be added to active pressures between the groundwater table and the bottom of the excavation.
5. Open cut height, h, should be limited to 8 feet.

B. Vertical Capacity of Soldier Pile



	(fs) Allowable Friction	(qa) Allowable End Bearing
CLAY FILL	0.3 KSF	$\frac{6.D}{B} \leq 40$ KSF
NATIVE SAND AND GRAVEL	2 KSF	

Recommended Minimum Embedment Depth 10 Feet below Base of Excavation

C. Anchor Pullout Resistance

	Friction (Adhesion)
CLAY FILL	0.3 KSF
NATIVE SAND AND GRAVEL	2 KSF

Verify with Load Test to 200% of Design Stress Level. See Text.

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Figure 2

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ATTACHMENT A
TIEBACK TESTING PROGRAM

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TIEBACK TESTING PROGRAM

Pullout tests and creep tests are required during construction to verify the design and proper installation of the tieback anchors.

Performance Test (200 Percent Design Load)

A minimum of two performance tests will be performed for each soil type that will be encountered in the bond zone of the anchors. Each performance test should be conducted according to the following procedure:

1. The geotechnical engineer will select the testing locations with input from the shoring subcontractor.

2. The performance test will measure anchor stress and displacement incrementally at loads up to 200 percent of the design stress. Load the anchor in increments of 25 percent of the design load (i.e., 0.25 DL, 0.50 DL, 0.75 DL, etc.) with each increment held for at least 5 minutes. Obtain and record deflection measurements during loading at intervals of 30 seconds, 1 minute, 2 minutes, 3 minutes, and 5 minutes. Measurements shall be made to an accuracy of 0.01 inch.

3. The maximum stress in the prestressing steel should not exceed 80 percent of the ultimate tensile strength during performance testing. The soldier piles and tiebacks that undergo performance testing may require extra reinforcement to permit stressing to 200 percent of design load as required for the performance test.

4. Perform a creep test at the 200 percent design stress reading by holding the load constant to within 50 psi and recording readings at 30 seconds, 1 minute, 2 minutes, 3 minutes, 5 minutes, and 10 minutes for temporary anchors. Continue with addition 10-minute increments if creep criteria are not met at the 10-minute interval.

5. A successful test is one that exhibits a linear or near-linear relationship between unit stress and movement over the entire 200 percent stress range, holds the maximum test unit stress without noticeable creep, and satisfies the apparent free length criteria. Noticeable creep is defined as a rate of movement of not more than 0.04 inch between the 1- and 10-minute readings, or not more than 0.08 between the 6- and 60-minute readings. If the reading does not stabilize to 0.08 inch or less per log cycle of time, the test shall be considered to fail the creep criteria. The minimum apparent free length (measured as the theoretical elastic anchor elongation under the applied load), based on the measured elastic and residual movement, should be greater than 80 percent of the designed free length plus the jack length.

6. Perform tests without backfill ahead of the anchor, if the hole will remain open, to avoid any contributory resistance outside of the anchor zone by the backfill. If the hole will not remain

open during testing, provide a bond breaker on the tie rods and backfill the no-load zone with a non-cohesive mixture.

Proof Test (130 Percent Design Load)

For each production tieback anchor, follow the proof testing procedures outlined below:

1. Load each anchor to 130 percent of the design load in increments of approximately 25 percent of the design load (i.e., 0.25 DL, 0.50 DL, 0.75 DL, 1.00 DL, and 1.30 DL). The maximum stress in the prestressing steel should not exceed 80 percent of the ultimate tensile strength during proof testing.

2. Hold each incremental load for a period long enough to obtain a stable deflection measurement while recording deflections at each load increment. Hold the 130 percent load for a minimum of 5 minutes, recording the movement at times of 30 seconds, 1 minute, 2 minutes, and 5 minutes.

3. A successful test is one that exhibits a linear or near-linear relationship between unit stress and movement over the entire stress range, holds the maximum test unit stress without noticeable creep, and satisfies the apparent free length criteria as indicated for the performance testing.

4. Typically, movement of the anchor in excess of about 3 inches is indicative of deficiencies in installation. Normally, total movement in excess of 12 inches is considered a failure requiring replacement. To determine if a replacement or supplement is required, the geotechnical and structural engineer should review total movement between 3 and 12 inches.

5. Following proof loading, lock off each tieback anchor to 80 to 100 percent of the design load, except as specified otherwise.

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Key to Exploration Logs

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance.
Soil density/consistency in test pits is estimated based on visual observation and is presented parenthetically on the test pit logs.

SAND or GRAVEL Density	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 – 4	Very soft	0 – 2	<0.125
Loose	4 – 10	Soft	2 – 4	0.125 – 0.25
Medium dense	10 – 30	Medium stiff	4 – 8	0.25 – 0.5
Dense	30 – 50	Stiff	8 – 15	0.5 – 1.0
Very dense	>50	Very stiff	15 – 30	1.0 – 2.0
		Hard	>30	>2.0

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, probably below optimum
Moist	Probably near optimum moisture content
Wet	Much perceptible moisture, probably above optimum

Minor Constituents

	Estimated Percentage
Not identified in description	0 – 5
Slightly (clayey, silty, etc.)	5 – 12
Clayey, silty, sandy, gravelly	12 – 30
Very (clayey, silty, etc.)	30 – 50

Legends

Sampling Test Symbols

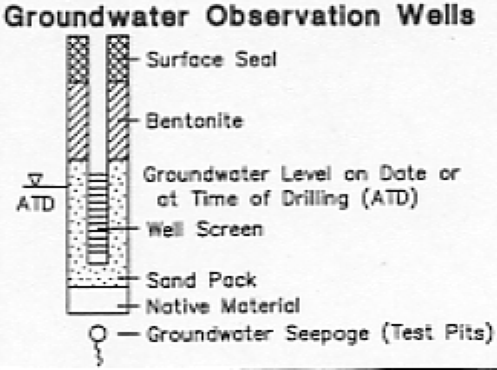
BORING SAMPLES

- ☒ Split Spoon
- ⧅ Shelby Tube
- ▥ Cuttings
- ◫ Core Run
- * No Sample Recovery
- P Tube Pushed, Not Driven

TEST PIT SAMPLES

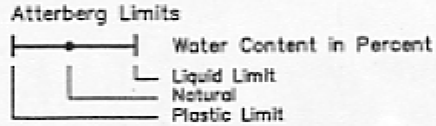
- ☒ Grab (Jar)
- ⧄ Bag
- ⧅ Shelby Tube

Groundwater Observation Wells



Test Symbols

GS	Grain Size Classification
CN	Consolidation
UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits



PID	Photoionization Detector Reading
CA	Chemical Analysis
DT	In Situ Density Test

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Figure A-1

Boring Log HC01-1

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 13

Depth in Feet

Depth (ft)	Soil Description
0	1.5 inches asphalt over medium stiff to stiff, moist, gray, sandy to very sandy, silty CLAY with wood debris.
15	Medium dense, moist, silty, gray, gravelly, fine to medium SAND.
	Petroleum-like odor.
	Very dense, wet, gray-brown, gravelly, silty to very silty, fine to medium SAND.
	Trace gravel from 23- to 29-foot depth.
30	Hard, wet, gray, slightly gravelly, fine, sandy SILT.

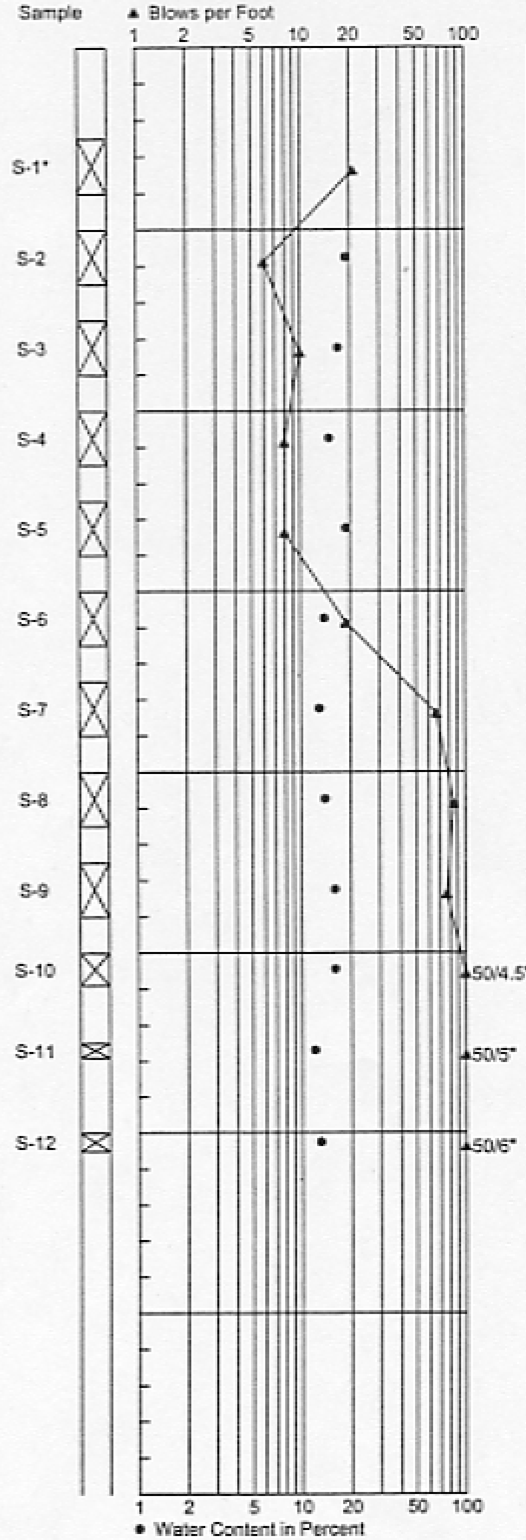
Bottom of Boring at 30.5 Feet.
Completed 09/18/01.

Datum: City of Seattle

∇ ATD

STANDARD PENETRATION RESISTANCE

LAB TESTS



BORING LOG 701902BL.GPJ HC_CORP.GDT 2/12/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-2

ATTACHMENT B
SHORING MONITORING PROGRAM

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The shoring wall should be monitored using controlled optical surveying of the elevation and horizontal position the top the soldier piles, with survey targets placed on about a 20-foot spacing (or every fourth pile). Monitoring should occur at least twice weekly, or daily if movements in excess of 1 inch are recorded at the top of the soldier piles. Monitoring should continue throughout the construction period, until the engineer has determined that readings can be stopped. Reported data should include date, total horizontal and vertical movement, relative to the initial baseline readings, and relative horizontal and vertical movements compared to the prior readings.

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